DESIGN AND CONSTRUCTION OF STEEL PLATE SHEAR WALLS

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ABSTRACT

With the first U.S. code for steel plate shear walls (SPSW) in the recently released 2005 AISC Seismic Provisions, the system will have a new level of exposure among design engineers in the U.S. There are several issues, however, that an engineer will face in deciding whether to use SPSW, and in the process of design. Aspect ratio limits in the provisions make SPSW impractical for low-rise, and shorter mid-rise structures unless between-floor stiffeners are used, but the use of between-floor stiffeners is not well defined. Plate material and thickness is of paramount importance in SPSW design, but the range of material or thickness that is appropriate is not clear. Three different methods for column design are presented in the provisions, but each comes up with different forces.

Also, in determining whether to use SPSW for a project, issues such as cost, constructability, column size, length of wall required, story drift and floor accelerations should be considered. Using a 2 story sample building design, it was found that SPSW yields a structure that uses 13% less steel than moment frames, considerably less field welding than moment frames or braced frames, and significantly less wall length than braced frames.

Introduction

Steel plate shear walls (SPSW) consist of a plate bounded at the sides by columns, also referred to as vertical boundary elements (VBE), and at the floor levels by beams, also referred to as horizontal boundary elements (HBE). The alternate nomenclature for beams and columns emphasizes the boundary elements role of resisting the tension field developed in the plate. Figure 1 shows an example of a SPSW that was constructed in San Mateo County, CA.

There has been a shift in the design methodology from the SPSW buildings built in the 1970’s to the SPSW buildings built in the past couple decades. Early designs for SPSW panels used stiffeners to preclude buckling in their relatively thick shear plate. As such, these walls are commonly called “stiffened”. In the early 1980’s research was spearheaded in Canada by Kulak, Thorburn, Driver, Grondin, and others to develop a design methodology that utilized the post-buckling capacity of web plates. It was found that, similar to shear in plate girders, the usefulness of a web plate is not limited to its buckling capacity. With boundary members

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designed to resist the forces of the web plate, not only does tension field action develop, but the tension diagonals can yield. Significant ductility and energy absorption has been noted in this type of wall, that has been referred to as “unstiffened” or “thin-walled” SPSW. It should be understood that the terms “stiffened” and “unstiffened” are a bit of a misnomer, however, as the essential difference in design methodologies is the degree to which a plate is allowed to buckle, rather than whether stiffeners are used. The web plate of the SPSW shown in Fig 1 is designed to develop tension field action at relatively low shear and would therefore be considered unstiffened. The 2005 AISC Seismic provisions and the rest of this paper only address unstiffened SPSW.

In the 1990’s considerable research on SPSW was done in the United States by professors Bruneau, Berman, Elgaaly, Caccese, Astaneh, and others. U.S. design provisions were, for the first time, published in the 2005 AISC Seismic provisions. Additionally, Bruneau and Sabelli are producing an AISC design guide on SPSW which is expected to include recommendations for seismic and non-seismic applications for SPSW. With these upcoming publications, SPSW will gain a new level of exposure with U.S. engineers. This paper is geared towards design engineers who might be considering using SPSW and contains:

1. Discussion of design considerations when using the 2005 AISC Seismic Provisions.
2. Comparison of SPSW to special concentrically braced frames and special moment frames.
3. Summary of reasons why the author used SPSW for 3 large residences in Northern California and experiences encountered in their design and construction.

**Design Considerations for SPSW**

The 2005 AISC Seismic Provisions refer to SPSW as special plate shear walls. The term special indicates that, like special concentrically braced frames or special moment frames, that special plate shear walls are expected to have increased ductility over their ordinary counterparts. While the AISC Seismic Provisions do not include ordinary steel plate shear walls, if they are included in future editions it can be conjectured that one of the differences may be simple shear beam-to-column connections (as opposed to moment connections required for SPSW).

In the process of SPSW design, the author has come across several issues that require...
judgment outside of what is presented in the AISC Seismic Provisions. Questions such as how thin a plate is appropriate, how slender a wall is possible, what plate material can be used, and how to interpret the code when it comes to column design are important in the design process, and are discussed below. First, a brief description of the design process is required to highlight some of the issues.

**Design Process**

To achieve the high level of ductility expected of SPSW, the boundary members have to be designed to remain elastic as the plate yields. As a result, size of beams and columns are designed based on the plate thickness. Plate design, however, is based on angle of inclination of the tension field, which is dependent on the boundary members. The design process, therefore, becomes iterative, but with closed form equations is well suited for a spreadsheet.

In order to begin the design process it is necessary to assume an angle of inclination, $\alpha$, for the tension field relative to the vertical. AISC Seismic Provisions give an equation for $\alpha$, that will need to be checked after boundary members are designed. The capacity of the wall and boundary member sizes will not be so sensitive to $\alpha$, however, that multiple iterations are usually necessary, unless drift controls the design. In fact, the equation for $\alpha$, is not entirely accurate. The equation is based on pinned beam to column connections with SPSW panels above and below. While $\alpha$ equations are available for other conditions, it was determined by the authors of AISC Seismic that these variations in $\alpha$ are not significant. An initial estimate of 40º will usually suffice.

After the plate is sized using the assumed angle, $\alpha$, the expected yield force of web plate that will be applied to HBE’s and VBE’s should be calculated. The expected yield stress of the web plate is $R_yF_y$. When reducing the expected yield stress into vertical and horizontal components, the fact that the length of plate on the angle is longer than the vertical or horizontal length should be taken into account to avoid overly conservative design. For instance, the vertical component of the expected yield stress to be applied to the HBE is $R_yF_y(\cos \alpha)(\cos \alpha)$ rather than just $R_yF_y(\cos \alpha)$. The upcoming AISC Design Guide by Bruneau and Sabelli discusses this further.

**SPSW Aspect Ratio and Intermediate HBE’s**

The length of an SPSW panel is limited by an aspect ratio of $0.8 < L/h \leq 2.5$. This allows for fairly wide walls, but not tall and slender walls. For instance, in the example building shown in Fig 4, the 15’ story heights would require a wall 12’ long. A 12’ long SPSW with 3/16” A36 plate will have a capacity on the order of 300 kips. The example building has a total static base shear of just under 300 kips, meaning that one SPSW panel could resist the entire shear for the building. Using only one lateral resisting element is not desirable for its lack of redundancy and lack or torsional resistance; several SPSW panels with less capacity are much more preferable.

The projects described later in this paper used intermediate HBE’s to reduce panel width. Reducing the width of SPSW panels serves two purposes – reducing the capacity to be more proportional to a low-rise building, and fitting SPSW panels into short lengths of wall. The same reasons could apply to the example building shown in Fig 4. The 7.5’ long SPSW panels have a capacity of just under 75 kips each which, with 4 walls in the building, gives a lateral resisting system with redundancy, torsional resistance, and fairly little wall taken up with lateral
resisting elements.

How to use intermediate HBE’s between floors is not well defined in the AISC Seismic Provisions. The commentary mentions that additional horizontal intermediate boundary elements can be used to adjust the aspect ratio of a SPSW panel, but the main body of the code requires that all HBE to VBE connections be moment connections and that all HBE to VBE intersections be laterally braced. It is the author’s interpretation that these provisions do not apply to intermediate HBE’s. This should be addressed, however, in future editions of the code.

With the use of intermediate HBE’s, the slenderness and capacity of a wall is perfectly adjustable. Walls as slender as the one shown in Fig 2 have been designed and constructed. A nonlinear pushover analysis was done on this wall to ensure that the boundary elements will remain elastic while the web plate yields.

**Web Plate Thickness and Material**

When sizing the web plate there are a few things to keep in mind. It is ideal to size the plate at each level to just satisfy the design shear force. This will make it more likely that the plate will yield at multiple floors and it will make for a more economical design of the columns.

At upper floors of a mid-rise or all floors of a low-rise building it becomes clear that even 3/16” to 1/8” plate can have more capacity than is needed. The question quickly becomes: how thin of plate should be used? AISC Seismic Provisions do not limit plate slenderness, instead, the commentary states that story drift limitations will indirectly limit the plate slenderness. For designs such as the ones shown in Fig 1 and Fig 2 the material and constructability may control instead.

As the web plate gets thinner the connection to boundary members becomes more difficult. The connection shown in Fig 3 is from a SPSW used for a high-end residence in San Mateo County, California. 14 gage plate was specified with continuous welds on both sides to the boundary members. Inadvertently, the steel fabricator shipped some of the panels to site with only intermittent welds around the panel perimeter. It was discovered that, while the horizontal welds were fairly easy to complete in the field, the welds along the VBE’s required special attention. Initial attempts at vertical welding presented some difficulty, when uphand welding (starting at the bottom and going up) began burning through the light gage plate. A special downhand (starting from the top and going down) procedure was developed by the fabricator and qualified by the special inspector. The resulting welds were found to be adequate, but this demonstrates some of the challenges when using light gage web plate.

Another factor to consider when using thin web plate is the shear that causes the plate to buckle. Plate buckling is expected to occur as the tension field develops, but it is not desirable
for service loads, e.g. a stout wind. Researchers who have run tests on SPSW’s report extremely loud banging as the loading reverses. Even though there haven’t been any documented cases of SPSW buildings making loud banging sounds in wind storms, SPSW are relatively new and it may yet become an issue. Theory of elasticity texts give formulas for calculating plate buckling and it is worthwhile to understand the load at which an SPSW will buckle. Introducing intermediate HBE’s with shorter panel lengths will increase the buckling load.

The third consideration when choosing plate thickness is the material specification. Section 6.1 of the AISC Seismic Provisions limits the materials that can be used in the seismic load resisting system. Acceptable plate material includes A36 or A1011 Grade 55 (in either SS or HSLAS). A1011 is hot rolled sheet steel and as such should have more pronounced yielding and more ductility than cold rolled sheet metal. It would be much more preferable, however, to use A1011 grade 30 or 33 sheet steel as it has a lower yield and more elongation. While the commentary discusses low yield point plate material, it is suggested that future editions of the AISC Seismic Provisions explicitly allow these materials.

Whenever light gage plate is used for the web plate it is the author’s opinion that it is worthwhile to require coupon tests to measure actual yield strength and percent elongation. Since the performance of the building is so directly tied to the plate material, the greater understanding of the primary energy dissipating element will easily justify the slight additional cost.

### Column Design

The main body of the 2005 AISC Seismic Provisions states that the required strength of VBE’s shall be based upon the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle, $\alpha$. To reach this level of force the beams are required to develop plastic hinges. The commentary describes three methods that can be used to design the VBE. The first, nonlinear pushover analysis, will give the most accurate understanding of the SPSW response. Axial loads and moments in the columns caused by web plate yielding and plastic hinging of the HBE can be used to design the columns to remain elastic. It is important to note that elastic modeling of SPSW will not capture the intent of the design process: to size VBE’s to remain elastic while the web plate yields. To design the VBE for yielded plate and plastic hinging of HBE’s requires consideration of the nonlinear response of the elements. Nonlinear pushover analyses, however, are not that common in most design offices.

The second method called combined linear elastic computer programs and capacity design concept (LE+CD) uses an elastic model of the wall, considering moment connections and...
web plate to determine the loads in VBE. The VBE axial force determined using the model is then amplified by $\Omega_0$, and combined with the effect of the web plate yielding for the story for which the VBE is being designed. This method seems inconsistent with the wording of the provisions, in that the axial force is multiplied by $\Omega_0$, but the VBE moments from plastic hinging of the HBE and from web plate yielding in panels above are not included. Since moments created by the web plate at only the floor in question are considered, this method could result in a VBE that becomes inelastic before the web plate has fully yielded.

The third method comes from the Canadian steel code (CAN/CSA S16-01). Overturning moments from the design lateral forces are calculated and then factored up to a level corresponding to the expected yield strength of the web plate. A distribution of overturning moment is then specified for which axial loads in VBE’s can be obtained. The local bending moments from web plate yielding are required to be factored up to the level of expected yield strength of the plate also. Affects of moments caused by plastic hinging in the HBE is not addressed, but since it is not specifically left out it should be included.

These three methods produce three different results. The disparity between these methods should be addressed in future editions of the code provisions if not in future research.

The AISC Seismic Provisions also require that HBE’s and VBE’s satisfy the weak beam/strong column requirements for moment frames without consideration of the webs. The moments in VBE’s caused by the web plate tension field are fairly severe. Similarly, the axial loads caused by the overturning moments from web plates yielding are quite high. If these loads are not considered in the weak beam/strong column check, then hinging in the HBE’s rather than the VBE’s is not ensured, and the usefulness of the provision is not clear.

**Comparison of SPSW to Other Lateral Systems**

With this added tool in our toolbox of lateral resisting systems, it is worthwhile to make a comparison between SPSW and some of the other choices available. A sample building was designed using special plate shear walls, special concentrically braced frames and special moment frames. A two story office building was selected that has 4-30’ bays in one direction and 3-30’ bays in the other direction. The two floor heights are each 15’. RAM software was used to optimize the gravity structure. Lateral bracing members for SMF and SPSW were included, but gusset plates, continuity plates, shear tabs, base plates and stiffeners were not considered in the summation of frame weight. Fig 4 shows an isometric view of each building.

**Cost Comparison**

An elevation of the SPSW design is shown in Figure 4. The design was carried out per the 2005 AISC Seismic Provisions with the some liberties taken, such as the addition of intermediate HBE’s and the use of ASTM A1011 grade 33 plate. Four SPSW panels were required in each direction. Some additional floor members were required to brace the SPSW at both columns and at both floors. The VBE design considered the moments and axial forces due to all web plates fully yielding and HBE’s developing plastic hinges.

The special concentrically braced frame was accomplished with HSS6x6x3/8 columns, HSS5x5x1/4 braces at the first floor and HSS4x4x1/4 braces at the second floor. Four braced frames were used in each direction, and each braced frame extended the entire length of one bay, 30’.
The special moment frame option was designed with two 2-bay frames in each direction. Laying out the frames so they don’t meet at the corners eliminated the need to consider combined orthogonal seismic forces. The columns are W14x159 and the beams at the first floor and roof are W24x62 and W18x40 respectively. The frames were drift controlled.

The weight of the structural steel was summed up for each design and is tabulated in Table 1. Weights were then normalized to the weight of the moment frame structure. The SPSW structure was 87% of the weight of the moment frame, while the SCBF structure was 79% of the moment frame. As would be expected both SPSW and SCBF result in less steel than the moment frame, but SPSW may mean slightly more steel than SCBF. Even though SPSW is similar to a braced frame with tension-only braces, the forces of the tension field on the boundary members causes some increase in their size over braced frames.

Stopping with just a steel weight comparison would not give the full picture of construction costs, however. One of the reasons that SPSW was used for the residence in Atherton, California (see later discussion) was that the panels came out of the shop fully constructed. The SPSW panels shown in Fig 4. could be shop constructed and erected without the need for field welding. Table 1 shows the number of field pieces and field welds required for each design. With 8% less pieces in the field and 256 less field welds, the SPSW option will be easier and faster to erect than the SCBF.

**Other Considerations**

The three SPSW projects that the author was involved with were all highly architectural buildings. Thickness of walls, length of solid walls and locations of solid walls were the driving force behind the choice to use SPSW. As shown in Table 1, SPSW columns can be smaller than moment frame columns and take up less wall length than SCBF.

Table 1 also includes inelastic drift, $\Delta_m$. The relative stiffness of the systems puts SPSW between a braced frame and a moment frame. When considering the relative performance of different systems it is important to examine not only drift, but also accelerations (Mayes et al 2005). A well known example of acceleration related damage is the Olive View Hospital which happened to be constructed with steel plate shear walls (Celebi, 1997). During the Northridge Earthquake, peak acceleration at the roof was recorded to be 2.31g while peak ground acceleration was 0.82g. The amount of damage to secondary structures and equipment including water damage from broken pipes greatly outweighed any structural damage.

The Olive View Hospital is different from typical SPSW designed per the AISC Seismic Provisions in two very important ways. As discussed in the introduction there has been a shift in the design methodology for SPSW. The Olive View Hospital used “stiffened” steel plate shear walls. That is, the 5/8” and ¾” thick web plate (Anon, 1978) was designed not to buckle. Additionally, in reaction to the failure of the previous Olive View Hospital in the Whittier Earthquake, high loads were used in an effort to reduce the amount of structural damage. It is obvious from the amount of acceleration amplification that the lateral resisting system remained virtually elastic during the Northridge Earthquake. If an SPSW plate thickness and configuration are designed to meet the seismic loads without significant design overstrength, then nonlinearities in the seismic response will reduce expected accelerations. It has been shown that moment frames, being drift controlled, usually exhibit less inelasticity than other systems (Mayes et al, 2005). As such, it can be expected that SPSW would likely have less acceleration amplification than the SMF option.
Figure 4 – Comparison buildings designed with special plate shear walls (a), special concentrically braced frames (b) and special moment frames (c). Elevation view of the SPSW design (d).

Table 1. Factors for comparing SPSW, SCBF and SMF lateral resisting systems.

<table>
<thead>
<tr>
<th>System</th>
<th>Cost / Construction</th>
<th>Design</th>
<th>Architectural</th>
<th>Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Steel Weight (kips)</td>
<td>% of SMF Wt</td>
<td># of Field Pieces</td>
<td>Resp. Mod. R</td>
</tr>
<tr>
<td>SPSW</td>
<td>173.8</td>
<td>87%</td>
<td>150</td>
<td>0</td>
</tr>
<tr>
<td>SCBF</td>
<td>158.1</td>
<td>79%</td>
<td>162</td>
<td>256 fillet</td>
</tr>
<tr>
<td>SMF</td>
<td>200.2</td>
<td>100%</td>
<td>182</td>
<td>96 CJP</td>
</tr>
</tbody>
</table>
The last column shown in Table 1 is meant to point out the repair options that might be available after an earthquake. In the braced frame and SPSW it would be possible to replace damaged components, while repair methods for moment frames would require additional work.

**Successful SPSW Projects in Northern California**

The author has been involved with three large residences where SPSW was determined to be the best option. All three are in the final phases of construction at the end of 2005. The reasons for using SPSW are described in the following list:

1. Speed of construction. One of the residences had a very short construction schedule. It was determined that fully shop fabricated SPSW would allow faster erection.
2. All three of the projects are highly architectural in that there are very few lengths of solid walls. It was necessary to fit lateral resisting elements into short lengths of walls or use moment frames. SPSW allowed the use of very slender elements (see fig 2).
3. Braced frames in tall slender configurations require large gusset plates. Configurations as slender as shown in Fig 2 would not be practical as a braced frame.
4. Thickness of walls was also important to the architects. Moment frames that would require thick walls or bump-outs to hide large columns was not an option.
5. As is shown in the preceding section, SPSW yields less structural steel weight and should therefore be cheaper than moment frames.

All three projects were designed before the AISC Seismic Provisions were available. The designs were carried out using the Canadian Steel Code (CAN/CSA 2001). 2d pushover models were used to verify the designs. 12 gage and 14 gage web plates were used and small angles were attached to the columns to provide a surface to which the plate was welded. Some type of fish plate or angle was needed to allow for tolerances in the plate and column. Care was taken not to attach anything to the web plates or to drill holes through the web plate for plumbing or conduit. A gap of approximately 1” was left between infill studs and the web plate in an effort to reduce damage from minor earthquakes that might buckle the plate.

**Conclusions**

Steel plate shear walls were an excellent choice of lateral systems for three large residences in Northern California. The high capacity possible in a SPSW panel that is relatively tall and slender helped satisfy the restrictive architectural requirements. A cost comparison shows that the design of a sample 2 story building required 13% less steel than a comparable moment frame and if detailed properly would not require field welding.

The 2005 AISC Seismic Provisions and the upcoming AISC Design Guide for steel plate shear walls will give SPSW a new level of exposure among practicing engineers. The following are suggestions for further research and future editions of the SPSW design provisions:

1. It should be defined whether intermediate HBE’s (between floors) require moment connections to the VBE’s. Also it should be stated that intermediate HBE’s do not require lateral bracing if the VBE is designed accordingly.
2. Further research should be conducted to determine what level of plate slenderness causes
buckling of the plate under service loads and if this results in audible banging or other negative effects.

3. The materials allowed for the web plate should explicitly include ASTM A1011 SS Grades 30 and 33. These materials have greater ductility than ASTM A1011 HSLAS Grade 55 which is already allowed.

4. There are three methods presented for designing VBE’s that give significantly different results. These variations should be reconciled.

5. The method for designing VBE’s called linear elastic computer programs and capacity design concept, considers axial forces amplified by $\Omega_o$, but moments due to web plate yielding in panels above or plastic hinging of the HBE’s is ignored. This appears inconsistent with the intent of the provisions, which is that VBE’s remain elastic while the web plate yields.

6. Strong column / weak beam design checks are specified to be carried out without considering web plate tension. Since a large portion of the axial forces and bending moments in the columns will be caused by web plate tension, this design check does not limit the plastic hinging to HBE’s.

References


Bruneau, Michel, Sabelli, Rafael and Pottebaum, Warren. 2006 AISC Design Guide for Steel Plate Shear Walls American Institute of Steel Construction (yet to be released).


